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Haley & Aldrich, Inc.

PRELIMINARY GEOTECHNICAL EVALUATIONS
FOR
THE PROPOSED DEVELOPMENT OF PIERS 1, 2 AND 3
SOUTH BOSTON, MASSACHUSETTS

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Consulting

Geotechnical Engineer

Geologists and

Hydrogeologists

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October 1984

Haley & Aldrich, Inc.

Consulting
Geotechnical Engineers,
Geologists and
Hydrogeologists

12 October 1984
File No. 5556

HBC Associates
c/o Carpenter and Company, Inc.
175 Federal Street
Boston, Massachusetts 02110

Attention: Mr. Harry Spence

Subject: Preliminary Geotechnical Study
Piers 1, 2 and 3
South Boston, MA

Gentlemen:

We are pleased to submit herewith four copies of our report on the preliminary evaluations of the geotechnical aspects of foundation design and construction for the project.

The work has been undertaken in accordance with our proposal dated 20 September 1984, and your subsequent authorization. Please refer to Section VI for a summary of our conclusions.

Thank you for engaging us to perform this work. We look forward to working with you and other members of the design team during subsequent design and construction phases.

If there are any questions regarding the content of this report, please do not hesitate to contact us.

Sincerely yours,
HALEY & ALDRICH, INC.

Mark X. Haley
Senior Engineer

David E. Thompson
Executive Vice President

MXH/DET/dmm/0554T

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I. INTRODUCTION

1-01. General

This report presents the results of our preliminary evaluation of the geotechnical issues related to development of Piers 1, 2 and 3 on Northern Avenue in South Boston. The 18.5 acre site extends easterly along Northern Ave. from the entrance of the Fort Point Channel to approximately Pier 4. The location of the project is indicated on the Project Locus, Figure 1. The development program studied was as detailed in the Master Plan report prepared by Hellmuth, Obata & Kassabaum, P.C. (HOK) dated October 1981. The 3-million square foot (sq. ft.) mixed-use development consists of a major hotel, low and high rise luxury condominiums, retail and office space, a marina, boat moorings, public parks and structured parking.

The geotechnical studies reported herein are based on subsurface data collected and summarized by Haley & Aldrich, Inc. during previous involvement with various projects for Mr. Anthony Athanas (Anthony's Pier Four Inc.). Some of the data compiled previously has been included in this report to provide the reader with background information and a general understanding of site and subsurface conditions.

This report will present a description of the site and subsurface conditions, a summary of the proposed Master Plan development, a discussion of geotechnical considerations relevant to the project and design development, preliminary recommendations of foundation types and probable costs for the geotechnical related items. The costs are presented as unit costs for typical construction operations or foundation elements.

1-02. Purpose and Scope

The purpose of this preliminary geotechnical evaluation was to provide input to the project planners for use during their review and feasibility study of the Master Plan site development. In addition, soil samples were obtained and chemical tests performed to provide data to initiate an assessment of the environmental issues related to site development. The scope of services performed included:

1. Review the available subsurface information to assess the major geotechnical issues related to the Master Plan development.

2. Research, collect and summarize the readily available information on site history and previous site usage to formulate an assessment of the presence of hazardous materials or oil as defined by the Massachusetts Oil and Hazardous Materials Release Prevention and Response Act (Chapter 21E). The results of this study are presented in a separate letter report.

3. Obtain representative soil samples from up to four locations on the site and have chemical analyses performed on these samples by a certified laboratory to test for the presence of hazardous materials.

4. Conduct preliminary geotechnical analyses for the various elements of the Master Plan development including: foundation types for high, mid and low rise structures; feasibility of below grade floor space; site filling, dredging and excavation; stability and design of seawalls and bulkheads; and economic considerations.

Because of the limited time period for these studies and limited involvement of other members of the design team at this time relative to the development, certain basic assumptions were made. These assumptions pertain to structural elements, including column loads and spacing, building height and ground floor elevation.

1-03. Limitations

This report has been prepared for the specific application to the Development of Piers 1, 2 and 3 on Northern Avenue in Boston, Massachusetts, in accordance with generally accepted geotechnical engineering practice. No other warranty, expressed or implied, is made.

The preliminary recommendations presented in this report are based, in part, on information from subsurface explorations and the details of the proposed development available to us at this time. The nature and extent of variations in the subsurface conditions between explorations will need to be further defined during subsequent design phases of the project. If significant variations are encountered, it will

be necessary to re-evaluate the recommendations presented in this report. In addition, as changes in the design and location of structures and development are made, a review and evaluation of our recommendations will be necessary.

1-04. Elevation Datum

The elevations presented herein are referenced to Boston City Base (BCB), wherein El. 0.00 (BCB) is 5.65 ft. below National Geodetic Vertical Datum (NGVD) formerly USC&GS Mean Sea Level Datum of 1929.

Tide tables and bottom soundings on many site or navigation plans are referenced to Mean Low Water (MLW) Datum wherein MLW datum is 1.07 ft. above BCB (El. 0.00 MLW = 1.07 BCB).

II. SITE AND SUBSURFACE CONDITIONS

2-01. Data Base

During previous Haley & Aldrich studies of the site area, various sources were researched to compile the readily available subsurface information. These sources included Massachusetts Department of Public Works, Port of Boston Authority and test boring results published by the Boston Society of Civil Engineers. In addition, test borings were conducted in 1970 for H&A in connection with a proposed development of Piers 1 through 4. Since that time, no additional test borings have been conducted within the site area. Locations of these previous explorations are indicated on Figure 2 including the data source and the year when the borings were conducted.

On 4 and 5 October 1984, 15 test pits were machine excavated within the site. The test pit logs and a location plan for these explorations are currently being prepared. The test pits were excavated to visually examine the near surface miscellaneous fill soils and to obtain soil samples for chemical and physical testing deemed necessary for environmental studies. Information derived from the test pits and the results of the chemical tests will be included in a supplemental report upon completion of the testing.

2-02. Site Conditions

The existing site is relatively flat with grades typically in the range of El. 17 to 18. The majority of the site is utilized for commuter parking except for the following areas:

- An existing one story warehouse structure located on Pier 1 and utilized by Anthony's Pier 4 Inc. as a warehouse and for cold storage;
- A small sandwich shop located along Northern Avenue;
- A commercial lobster company utilizing the end of Pier 2 for a storage and mooring area;
- An area where old railroad cars are currently being stored along the two original railroad spurs on Pier 1; and

- An area between Piers 1 and 2 where a large stock pile (15 to 20 ft. high) of granite blocks has been placed.

A near vertical face granite block seawall defines the water's edge for most of the site area. The seawall along the curved portion of Pier 1 appears to be in good condition with only a slight seaward lean apparent. The seawall at the end of Pier 2 for that portion observable typically appears to be in good condition. A small section, approximately 10 ft. long near the intersection of Pier 2 and the Pier 1-2 slip was observed to be in need of minor repair. The seawall along the east side of Pier 2 moved significantly outward and downward in the late 1960's. This wall was stabilized in the early 1970's by replacement of the backfill. The wall is considered to be marginally stable in its present condition.

Vertical seawalls do not exist in two areas recently filled. One area is at the end of Pier 2 between Piers 1 and 2 where a brick and concrete rubble dike was constructed during the 1969 filling. The dike consists of a sloped face with random large granite blocks. This area will probably require remedial work during development of the site. A second area is along the water's edge between Piers 2 and 4 that was filled in 1971. The shore protection in this area consists of a sloped, placed granite block facing. The slope and the facing are in good condition.

The historical data relative to site filling and past site usage is included in a separate letter report prepared by H&A detailing the Oil and Hazardous Material Site Evaluation.

With regard to planning future site development, it should be noted that old wharf and pier structures (seawalls, piles, etc.) exist within the filled land area. In addition, old building foundations and underground utilities are expected to be present.

2-03. Subsurface Conditions

The available data from subsurface explorations indicate the following soil and rock sequence in order of increasing depth below ground surface.

- Miscellaneous Fill
- Organic Soils
- Marine Deposits
- Outwash Deposits
- Glacial Till
- Bedrock

The above sequence reflects the general order of deposition of the soils above the bedrock. One or more of the units may be absent at specific locations within the site area. The soil strata encountered are discussed in the order of deposition including the bedrock as follows:

Bedrock

The bedrock underlying the site is known locally as Cambridge Argillite. The bedrock is typically found to be highly fractured and weathered near the surface becoming moderately hard and fractured with depth. The data indicate that a zone of highly fractured weathered rock may extend to a depth of 20 ft. or more. The depth to bedrock ranges from 130 to 150 ft. below existing ground surface. Refer to Figure 3 for "Top of Bedrock Contour Plan".

Glacial Till

The advance of the glacial ice sheets over the New England Area during the Pleistocene Epoch resulted in the deposition of an unsorted mixture of clayey sand and gravel to sandy clay typically referred to as glacial till. This material, deposited over the bedrock surface, has been found to contain numerous cobbles and boulders. The glacial till soils are typically classified as dense to very dense. The top of the glacial till stratum varies from El. -95 to El. -120 (112 to 137 ft. below ground surface) across the site. Refer to Figure 4 for "Top of Glacial Till Contour Plan".

Outwash Deposit

As the glacial ice melted and the edge of the glacier retreated, soil frozen within the ice mass was carried by meltwater streams and deposited as a stratum of sand and gravel. The outwash deposit overlying the glacial till varies in thickness from 0 to 20 ft. The outwash deposit was found to be dense to very dense based on standard penetration resistance.

Marine Deposit

A thick deposit of silty clay known locally as "Boston Blue Clay" overlies the glacial till and outwash sand. This stratum was deposited following the retreat of the glacial ice sheets and a rise in the sea level that inundated the site. The fine grained materials comprising the majority of this layer settled out of suspension in the calm waters of the Boston Basin. The deposit, approximately 90 to 100 ft. in thickness, consists of medium stiff to stiff gray silty clay. Near the surface of the deposit the clay becomes very stiff as a result of desiccation during times of lower sea level. A thin layer of silty fine sand typically overlies the clay except in areas where dredging has occurred.

The surface of the marine deposits varies considerably due to alterations at the site from past usage. Previously existing channels and boat slips are evident based on the test boring data and bottom soundings prior to recent site filling in early 1970. The elevation of the surface of this stratum is contoured on Figure 5 and varies from approximately El. -10 to El. -35.

Organic Soils

Following deposition of the clay soils during the glacial age, two different types of organic deposits formed throughout the harbor area:

- Organic silt with shells, which was deposited in salt water by tidal action; and

- Salt marsh peat, which accumulated along the shore line of a slowly rising sea.

The organic silt and peat are typically soft to medium stiff, with low strength and high compressibility.

Miscellaneous Fill

With development of the harbor area during and following the 17th century, many waterfront structures were built out into the harbor. The site area was filled during the 1860's and 70's for development of harbor facilities and filling of the South Boston tidal flats.

The fill can be expected to consist of an unsorted mixture of sand, gravel, silt, clay, wood, brick, cinders, glass, granite blocks, etc. Numerous wood piles that supported old wharves and other structures can be expected to occur throughout the site. Based on historic records for the site, it is believed that the initially placed fill (El. 0 to El. 12) at Piers 1 and 2, consists of dredge spoil excavated from near the mouth of the Fort Point Channel. The upper 5 ft. (El. 12 to El. 17) of the fill consists of more granular soils most probably placed as a cap over the softer dredged soils. Consequently, the original fill soils will consist of a variety of soil types intermixed as a result of the dredging and placement process.

The recent fill materials placed during the late 1960's and early 1970's between Piers 1 and 2 and between Piers 2 and 4 consist primarily of inorganic mineral soils excavated from building construction sites throughout the Boston area. Boulders greater than 12 inches in diameter, organic soils, wood, trash, and other deleterious materials were precluded from the fill by the specification for the work. Although the wood decking, stringers and pile caps of the existing wharfs were removed prior to placing the fill, the piles and old seawalls between Piers 1 and 2 were buried. The locations of these buried structures were surveyed prior to placement of the fill and are recorded on drawings for future reference.

In summary, the miscellaneous fill soils can typically be characterized as fine grained soils consisting of sandy clays or silty clays with zones or layers of organic or granular soils.

Generalized soil profiles are presented on Figures 6 and 8 and summarize subsurface conditions as interpreted from available test boring information. These profiles were prepared in 1970 and are included herewith to provide background information.

2-04. Groundwater Conditions

Water levels throughout the site will respond to tidal fluctuations. The tidal response at a specific location will depend on the permeability of the soils between that location and the source of free water. It can be expected that the response to tidal action will vary considerably across the site due to the complex nature of the near surface fill soils.

In addition to normal or static water levels at the site, perched water or areas of trapped water will also be encountered. This water, typically resulting from rainfall and surface runoff, will need to be considered if near surface excavations are proposed.

Water levels at the site will also be influenced by inflow and outflow from nearby storm drains or sewage lines especially those discharging into the harbor.

Groundwater levels at the site should be expected to reflect tide levels in Boston Harbor typically ranging from El. 1.1 Mean Low Water (MLW) to El. 10.5 Mean High Water (MHW).

For design purposes, consideration must be given to storm tides and the probability that below grade structures (particularly those near the site perimeter) will experience groundwater levels equal to the storm tide level. A design water level of El. 16.0 should be assumed for below grade structures, unless the design provides for a groundwater cutoff and underdrainage.

2-05. Tide Levels

The U.S. Army Corps of Engineers has developed data on observed tide levels in Boston Harbor and the predicted tide level frequency of occurrence. The following tide level frequency data are estimated by the Corps for data through August 1979.

FREQUENCY IN YEARS	HIGH TIDE ELEVATION (BOSTON CITY BASE)
1	13.55
5	14.35
10	14.75
20	15.15
25	15.55
50	15.65
100	15.95
200	16.35

NOTE: El. 0.0 Boston City Base is 5.65 ft. below National Geodetic Vertical Datum (USC&GS MSL of 1929)

Lowest site grades are normally designed at El. 15.0 in the Boston area and the lowest recommended design floor level to avoid tidal flooding is normally taken as El. 16.0 although El. 17.0 is preferred.

III. PROPOSED DEVELOPMENT

3-01. General

The development plan utilized as a basis for this geotechnical evaluation is as presented in a report prepared by HOK, dated 20 October 1981 and entitled "Piers 1. 2. 3. Boston". The Master Plan Development provided planning and design concepts for a mixed-use development on the 18.5 acres of waterfront property along Northern Avenue in South Boston, Massachusetts. A plan indicating the proposed development is included on Figure 9. The 3-million square foot development program includes the following:

- 150,000 sq. ft. of office space;
- 900 to 1,000 condominium units at 1250 to 2400 sq. ft. per unit;
- 50,000 to 100,000 sq. ft. of retail space;
- a 1,000-room hotel with associated ballroom and convention facilities;
- structural parking for 2300 cars; and
- a 94,000 sq. ft. boat marina.

The Master Plan presented a site development concept with only an indication of building height and massing. In order to conduct our evaluations, basic assumptions of building height, massing, floor areas, column spacing and typical foundation loadings were assumed in order to assess foundation requirements and design and construction considerations.

The following sections will discuss our understanding of the Master Plan development and our basic assumptions.

3-02. Alterations to Existing Site Grades

Two major site alterations are indicated in the Master Plan. First, the excavation and creation of approximately 3 acres of open water for a marina within the central portion of the site is planned. Secondly, the filling of approximately 2 acres of open water adjacent to and east of Pier 2 is proposed.

For purposes of this study, the excavation level for the marina has been assumed to be El. -10 thereby providing approximately 10 to 11 feet of water at low tide. The level of the new fill was assumed to be in the range of El. 17 to

18 similar to the ground surface grades existing throughout the remainder of the site. Based on these assumptions it is estimated that approximately 130,000 cubic yards (cy) of excavation for the marina will be required and approximately 130,000 cy of fill will be required for land reclamation.

No other major grade changes within the site have been anticipated. It is assumed that final site grades for roads, walkways and open space will be between El. 17 and 18.

3-03. Proposed Buildings

Various distinct building types are indicated in the Master Plan report. For purposes of this study the different components have been assumed to consist of the following:

Building	Approximate Plan Area (sq. ft.)	Assumed Building Height (stories)
A. Hotel Complex		
Hotel	63,000	11 to 30
Ballroom	63,000	3
B. Residential		
Towers	175,000 total plan area	38
Townhouses		2 to 5
C. Office	18,500	12 to 15
D. Retail	15,000(1)	1 to 2
E. Structured Parking	900,000 total floor area (2)	

(1) This retail space will occur in three small independent buildings. Additional retail space will occupy portions of the hotel, residential and office buildings.

(2) Structured parking will occur adjacent to or below the various buildings. The exact location and geometry of these structures are unknown at this time. The geotechnical issues relating to below grade parking are discussed in Section 4-03.

IV. GEOTECHNICAL CONSIDERATIONS

4-01. General

This preliminary geotechnical study has considered the building layout and structural configuration based on the Master Plan Report prepared by HOK. As further information is developed by the Architect and Structural Engineer concerning design layout, column loadings, bay spacing, total building loads, and other details, the preliminary recommendations presented herein will need to be reviewed and revised as required.

4-02. Site Development Planning

During future planning for the various structures and site development, existing site and subsurface conditions should be considered. The site is entirely composed of reclaimed land; the majority placed in the late 1800's and the remainder in the 1960's and 1970's. Granite block seawalls or rock and rubble dikes provide the shore protection at the water's edge. Within the site area, due to past site usage, below grade structures, timber piles, seawalls, foundation elements and utilities will be encountered. For these reasons the following preliminary criteria for site planning are recommended:

A. Building Setback Requirements at Existing Seawalls and Shore Protection

Proposed buildings or other structures are recommended to be set back a minimum of 20 ft. from the exposed face of the vertical seawalls. In areas where rock and rubble dikes exist, a setback distance of up to 90 ft. will be necessary. These setbacks are required to facilitate installation of new foundation elements and to reduce the impact of new construction on the existing seawalls.

Previous experience in the Boston Harbor area with similar granite block seawalls indicates that the walls have a marginal factor of safety. The potential exists for only a small change in loading causing a wall failure. For this reason, the support of any additional vertical load on the walls is not recommended.

Along the east side of Pier 2, the existing wall has undergone significant lateral and vertical movements in the past and will require reconstruction. Therefore, development should consider the type of shore protection required to augment the intended land and water activities proposed.

B. Site Utilities and Pavements

The site, being underlain by a thick deposit of clay, organic soils and having been filled in the past, is potentially susceptible to ground movements and surface settlements. The areas filled during the mid to late 1800's for all practicable purposes are not expected to undergo appreciable future settlement unless additional surface loading occurs by raising grades or adding structural loads. The areas filled in the early 1970's have a higher potential for future ground movements and this fact should be considered during development planning.

It is recommended that, where possible, soil supported site utilities be located within the limits of the areas filled in the 1800's. In the 1960 and 1970 fill areas, soil support of utilities is possible but additional attention to detailing connections, slope requirements and invert grades will be necessary. Within areas of the site where significant filling is proposed, large settlement can be anticipated and soil support of utilities are probably not possible. Further evaluation of design details for utilities and pavements in this area will be required.

In the area of the existing seawalls or other shore protection elements, the potential for loss of ground through the walls due to tidal action is present. Installation of a filter system (geotextile or graded soil filter) behind the walls is recommended to reduce the potential for loss of ground.

The near surface fill and organic soils will tend to compress and settle with time. Therefore, it is recommended that pavements and final surface treatments be designed to accommodate future ground movements.

C. Site Filling

As indicated in the previous sections, the subsurface soils will consolidate and settle if the soils are subjected to additional surface loads. Therefore, to the extent possible (considering flooding associated with storm tides), existing site grades should be maintained.

If site filling greater than 1 to 2 ft. is proposed, further evaluation of anticipated movements will be required. In addition, recommendations relative to design of pavements and utility support, as well as building foundations in these areas would need to be studied in greater detail.

As currently proposed in the Master Plan Report, approximately 2 acres of land will be reclaimed from the sea by filling a portion of the slip between Pier 2 and Pier 4. Fill heights in this area will range from 40 to 45 ft. The placement of this fill, approximately 130,000 cy in volume, will result in consolidation of the compressible underlying materials. This process of consolidation, especially for the thick layer of Boston Blue Clay, will occur slowly over time and result in significant surface movements for at least 20 years following site filling. Design of structures located within this area will need to consider the effects of the anticipated movements. End bearing piles or caissons for support of the buildings will need to be designed either to prevent down-drag forces from acting on them or to carry the additional drag load. In either case, a premium cost for foundations located in this reclaimed area will occur.

In addition to the geotechnical design issues related to filling a section of the harbor, numerous environmental issues must be considered and the various permits obtained prior to filling. At this time, it cannot be determined if on-site excavated material can be utilized for fill. Chemical and physical testing is currently underway to determine if the on-site soils are feasible for use as underwater fill.

Based on discussions with local excavation contractors, it was determined that the cost of trucking fill material from off-site and end dumping it in the fill area would presently range from $7.50 to $8.00 per cy. The material would consist of a clean sand and gravel.

D. Site Excavation

As currently proposed, a marina will be constructed within the central portion of the site. Construction for the approximately 3 acre marina will involve the excavation and removal of approximately 130,000 cubic yards of soil. Excavated soils will be comprised primarily of the miscellaneous fill including sand, clay, organic soil, ash, cinders, and miscellaneous debris in varying quantities. Below grade structures (seawalls, timber piles, etc.) will also be encountered during excavation and will need to be removed.

Use of this material for on-site filling in open water or within a containment structure would result in the placed fill having a loose consistency and subject to future consolidation and large ground movements. In addition, environmental issues relative to use of this material for land reclaimation within the harbor would have to be addressed.

Therefore, for current planning it should be assumed that the excavated material will be disposed of off-site. The material will need to be trucked to a DEQE approved landfill for disposal. Excavations for below grade building areas, if incorporated into the project, will encounter similar soil conditions and will require off-site disposal in a similar approved landfill.

The estimated cost for excavation and off-site disposal of materials from this site would presently be in the range of $10.00 to 12.00 per cy.

4-03. Building Foundation Systems

The selection of the most technically and economically feasible foundation support system is in part determined by the building geometry, column loadings, and subsurface soil conditions. Based on the available soil data, the near surface soils are not considered suitable for support of new structures. Building loads must be supported below the miscellaneous fill and organic soils. The buildings, (assuming no below grade floor areas) will require deep foundations. If two or three basement levels are constructed, use of a structural mat or individual spread footings bearing on the clay may be possible.

The following sections will discuss and provide additional details concerning the recommended foundation types.

A. Deep Foundations

As indicated above, the use of deep foundations, (piles or caissons) gaining their support below the unsuitable near surface soils, are recommended for new buildings having no basements. Various alternative pile types and caisson configurations are technically feasible.

Based on assumed building loadings, and the subsurface conditions present, an economic analysis of the various pile and caisson types was conducted. Currently, pre-cast, pre-stressed concrete piles will provide the most economical foundation element for most of the proposed structures. These piles would be driven to end bearing in the glacial till or to bedrock. Typical design loads for this pile type are 134 tons/pile (14" square) and 175 ton/piles (16" square). Based on 1984 construction costs and a typical subsurface profile at the site, it is our opinion that the probable cost for this pile type would be in the range of $30/ton of supported load. This cost does not include the cost of pile caps, any contingency, nor the general contractor's normal mark-up.

B. Mat and Spread Footing Foundations

The use of a mat or footing foundations bearing on the clay may be possible and economically feasible if multiple basement levels are incorporated into the project. The depth of excavation to make these types of foundation elements feasible must be deep enough so that all of the miscellaneous fill and organic soils within the building limits are removed. Based on the contours of top of inorganic soil (Figure 5) the excavation depth required would be in the range of 27 to 37 ft. for the majority of the site. The most advantageous area for basement excavations is along Northern Avenue where the thickness of the unsuitable bearing soil is the least and the distance from the harbor the greatest.

Post construction settlements of the mat and individual spread footings will need to be considered during future design studies especially potential differential movements between low, mid and high rise portions of the

structure. In addition, design of the lower floor slab and waterproofing considerations will need to be evaluated.

The cost of a mat or individual spread footings is primarily related to the cost involved in making the excavation and providing temporary earth support during excavation. The depth and size of the excavation therefore becomes the determinent for providing an estimate of cost. At this time, these items have not been defined in sufficient detail to provide a reasonable estimate.

C. Below Grade Building Areas

If below grade building space is incorporated into the project, basement walls and floor systems must be designed to resist hydrostatic forces. In addition, waterproofing of all below grade space will be required. Alternatively, a combination of waterproofing and pressure relief by underdrainage may be considered.

Other factors to be considered include design lateral earth pressures, temporary earth support during excavation, aspects of temporary and permanent dewatering and constructability. Further geotechnical assessments related to below grade space can be provided in greater detail when a decision as to the depth and plan area of possible basement space is made.

4-04 BULKHEADS, SEAWALLS AND SHORE PROTECTION

A. General

This section of the report discusses alternative types of bulkheads, seawalls and shore protection considered technically feasible for the proposed marina and along the perimeter of the proposed area to be filled east of Pier 2. The alternative structures to be discussed include the following:

- Anchored bulkheads
- Seawalls (pile supported and stone dike supported)
- Sloped shore protection

The alternative structures are discussed in relation to the type of environment in which they will be constructed, i.e. on land (marina) or in water (filled

land). All types of walls considered for the marina would be constructed prior to general excavation for the marina and site flooding. Placement of soil or stone berms in water is expected to be a common aspect of construction for all wall types considered for the filled land area.

Schematic details of alternative anchored bulkheads considered are included on Figure 10. Details of seawalls and sloped shore protection are indicated on Figure 11.

Construction for the marina will be discussed followed by construction for the filled area. A comparison of various design and construction alternatives, and cost factors for anchored bulkheads, seawalls and sloped shore protection is presented on Figure 12. Factors considered include cost, constructability, local construction practice, foundation set-back requirements for adjacent structures and environmental, corrosion, architectural and mooring considerations.

It should be noted that there are other possible types of structures or variations to the alternatives presented herein, which are not discussed. It is the intent of this section of the report to present a range of practicable alternatives for the proposed construction and to provide a general discussion of the design issues and relative merits of the alternatives presented.

The various alternative types of shore protection can be separated into two general geometric categories; sloping face or vertical face structures. Each type has certain advantages and disadvantages.

The major disadvantage of sloped shore protection is that it significantly impacts water access unless a pile supported platform or marginal wharf is constructed which extends beyond the sloped portion, allowing boat mooring and water access. In addition, the width of the sloped structure from toe to crest is usually large, thereby requiring adjacent building foundations to be set back at a relatively large distance from the water's edge. The advantage of this type of shore protection is the simplicity of construction and relatively low cost.

Vertical shore protection is advantageous in that it allows water access and boat moorings to occur immediately adjacent to the face of the wall. In addition, adjacent buildings can be located relatively close to the wall. The disadvantages of vertical shore protection are the more complex construction requirements and relatively higher cost compared to sloped shore protection.

These and other factors must be considered during site development studies to assess the geometric constraints among the proposed buildings, marina and shore protection. Additional discussion of the various alternates and design issues are included in the following sections.

B. Alternative Structures for the Proposed Marina

The proposed marina will consist of an irregularly-shaped, open water area to be located within the central portion of the existing land mass with connections to the harbor at the outboard end of Pier 2. Refer to Figure 9.

It is assumed that the finished surface grade adjacent to the marina will be approximately El. 17 and that the excavation or dredge level of the marina will be approximately El. -10. Thus, an excavation of about 27 ft. will be is required. It is anticipated that the majority of marina construction can be conducted from the land and the excavation made in-the-dry.

1. Anchored Bulkheads

Three types of anchored bulkheads are considered feasible for marina construction. The primary difference among the three types lies in the type of wall system. Secondary differences include method of wall installation, wall-tie rod connections, and the final visual appearance of the completed wall. The wall types are interlocking steel sheet piling, cast-in-place concrete diaphragm wall and precast concrete panel wall. Considering an estimated bulkhead height of 27 ft. (El. 17 to El. -10) and anticipated subsurface conditions, a typical anchored bulkhead design would consist of the following:

- 45 ft. deep wall (exposed 27 ft. wall height);
- a single anchor level (tie rods) at a depth of approximately 25 percent of the wall height, or about 7 ft. below ground surface,
- a deadman located about 35 ft. behind the wall.

Refer to Figure 10 for schematic details.

Construction of the bulkhead would occur prior to excavation for the marina and the construction methodology would be similar to that required for a typical braced excavation. The sheet piling would be driven in sequence to create a continuous wall. The concrete diaphragm wall and precast panel wall would be installed using the slurry trench method.

The available subsurface information indicates that the surficial fill is relatively free of large materials that could potentially obstruct bulkhead installation. Therefore, it is anticipated that, in general, driving of steel sheet piling or excavation of slurry trenches for concrete diaphragm or precast concrete panel walls can be accomplished without major difficulty. There will be locations along the bulkhead line, however, where pre-excavation of obstructions, such as existing seawalls or removal of wood piles, will be required. The extent of obstruction removal is anticipated to be small in relation to the total construction effort needed for wall installation.

Once the wall system has been constructed, excavation within the marina could begin. Anchorage (tie rods and deadmen) would be installed behind the bulkhead once the excavation within the marina reached the anchor level.

Steel sheet pile bulkheads are, by far, the most common form of anchored bulkhead constructed. Although not used locally for shore protection, concrete diaphragm walls and precast concrete panel walls have been used in other areas of the country. The construction methodology for concrete diaphragm wall and precast concrete panel walls has been used extensively for recent deep excavations in the Boston area and these wall types are believed to be feasible alternatives to steel sheet pile bulkheads.

All three wall types would have essentially the same impact on foundations of proposed adjacent buildings or structures. Foundation elements for adjacent structures could be located up to about 5 ft. from the bulkhead and must avoid anchorage system elements.

Comparing costs, steel sheet pile bulkheads are less costly than concrete diaphragm walls and precast concrete panel walls. However, concrete diaphragm walls and precast concrete panel walls are advantageous in that they are more corrosion resistant. Steel sheet pile walls are susceptible to corrosion, especially in the splash zone and consequently would have limited life compared to concrete walls.

Architectural considerations may require certain finishes on exposed portions of the wall. The precast concrete panel wall would have a smooth finish. The concrete diaphragm wall would have a rough finish, the roughness being a function of the quality of excavation and the type of material through which the excavation was made. The steel sheet pile wall will have an irregular finish due to the "Z" shape of the steel section and it will rust.

2. Pile Supported Granite Block Seawalls

This type of waterfront structure would consist of a timber pile-supported, granite block seawall. The blocks that form the seawall would rest on a crib and pile cap supported by timber piles. These walls would be similar to the seawalls that currently exist at the site.

The seawall would be designed as a gravity retaining wall (wider at the bottom than at the top) to resist the horizontal forces from the retained soil. The piles supporting the wall would be driven through the fill and organic soils and derive support through friction within the underlying marine clay. The height of the granite wall would be approximately 27 ft., from ground surface to dredge level. Refer to Figure 11 for schematic details.

Many seawalls have been constructed in the past similar to the type described. Today however, such seawall construction is not common especially of the height considered, primarily for reasons of cost.

Granite block seawalls are advantageous in that they will permit mooring of vessels adjacent to the wall, provide for easy water access, are non-corrosive and are natural in appearance. Foundation set-back requirements for adjacent structures would be greater than for anchored bulkheads wherein foundation elements would have to be installed about 20 ft. behind the exposed face.

The main disadvantage of the seawall is cost. It is our opinion that this alternative will cost significantly more than any of the other bulkhead alternatives.

3. Sloped Shore Protection

This structure would consist of facing an excavated soil slope to prevent future erosion from wave and tidal action. A filter material and rock fill would initially be placed over the excavated soil slope to provide a bedding for the armor or facing stone. Refer to Figure 11 for details.

This type of shore protection has the advantage of simplicity of construction, is non-corrosive, natural appearance and of relatively low cost. On the other hand, the sloped shore protection scheme would probably require construction of a pile-supported deck to permit mooring of vessels.

Regarding impacts on proposed adjacent structures, foundation elements would need to be installed beyond the crest of the soil slope which may extend 80 ft. back from the toe of the slope.

It is our opinion that sloped shore protection will be comparable in cost to a steel sheet pile bulkhead. Probable costs for sloped shore protection, including required pile-supported decking for water access, would be greater than all three bulkhead alternatives but less than the granite block seawall. Refer to Figure 12.

C. Alternative Structures for the Proposed Area to be Filled

Part of the proposed development will consist of filling an area approximately two acres in size, located

immediately east of existing Pier 2 and north of an existing sloped riprap wall between Piers 2 and 4. Refer to Figure 9.

Available subsurface information indicates that along the perimeter of the proposed fill, the mudline elevation varies from approximately El. -6 to El. -30 and the top of clay varies from approximately El. -10 to El. -35. From the mudline to the top of clay is a layer of soft bottom sediments. Construction of most types of walls to retain the fill would require removal of the soft sediments either by displacement berms or dredging.

For this study, it has been assumed that open filling in the water will be permitted and that a containment structure will not have to be constructed prior to fill placement. If open filling is not allowed due to environmental restrictions, a cofferdam structure will need to be constructed to contain the fill. Details of this type of structure can be provided when environmental criteria have been defined.

The following sections discuss the alternative wall types considered practicable for construction at the perimeter of the proposed fill area.

1. Anchored Bulkheads

   It has been assumed that the wall would be installed from a soil berm placed through water. It is anticipated that the berm would be advanced from the land or in conjunction with filling of the proposed area to be developed. The berm would consist of granular soils and would be placed in a manner to displace the majority of the soft bottom sediments. It is assumed that 4H to 1V granular fill berm slopes could be achieved in water. The berm would be built to near final site grade to facilitate conventional construction procedures for the wall construction. Refer to Figure 10 for schematic details.

   Of the various types of anchored bulkheads, only a steel sheet pile bulkhead is considered practicable for construction in the fill area. Because of the anticipated loose density and high permeability of the soil berm, slurry trench excavation techniques are not considered feasible for concrete diaphragm and precast concrete panel walls.

The stability of the soil berm placed through water is of concern, especially during construction, when equipment will need to operate on the berm crest. Future settlement of the fill soils, due to consolidation of the underlying clays, will also affect the bulkhead wall and anchorage system performance.

Corrosion consideration and foundation set-back requirements for adjacent structures are the same as for a steel sheet pile bulkhead constructed for the marina. The sheet pile wall will be subject to corrosion and will have a limited life span. Foundations of adjacent structures can be located about 5 ft. behind the wall and must avoid the bulkhead anchorage system.

It is our opinion that a steel sheet pile anchored bulkhead installed through the fill berm would probably cost three times more than a similar bulkhead installed on land for the marina, Figure 12. Primary factors include the cost of placing the berm, the longer and heavier sheet pile section, and larger deadmen.

2. Pile Supported Seawalls

This alternative would be essentially the same as that constructed for the marina. The required wall height would be approximately 27 ft. (El. 17 to El. -10). Because the depth to the top of inorganic soils (average El. -25), would be below the bottom of the seawall, fill would be required from the top of clay to the bottom of the wall. Alternatively, the wall could be built to bear on the inorganic soils below the mudline, but the cost would be prohibitive.

It is anticipated that a granular soil berm placed to El. -10 would be advanced along the seawall line, displacing the loose bottom sediments. This would be followed by installation of a steel sheet pile cofferdam, driving of wood piles through the berm, construction of pile caps and cribbing and placement of granite blocks for the seawall. The cofferdam would permit placement of the caps and construction of the wall to be conducted in-the-dry. The sheet pile cofferdam would be removed following wall construction.

This alternative would have relatively minimal impact on adjacent structures in that the new structure foundations could be set-back about 20 ft. behind the exposed face of the wall. The wall will experience vertical and horizontal movements due to long-term consolidation of the clay stratum. These settlements will result from the weight of the wall and from placement of approximately 40 ft. of fill over the clay. This wall would be non-corrosive and natural in appearance.

It is our opinion that the cost for this alternative would be greater than a steel sheet pile anchored bulkhead, Figure 12.

3. Stone Dike Supported Seawall

This alternative differs from the pile-supported seawall in that the foundation of the seawall would consist of a stone dike rather than a pile foundation and a soil berm. Similar to the granular soil berm discussed above, a stone dike would be advanced along the seawall line, displacing the loose bottom sediments and would be built to approximately El. -10. The seawall would then be constructed within a sheet pile cofferdam or through the water.

The dike would impact on foundations for adjacent buildings to a greater extent than a pile supported seawall. The toe of the dike may extend about 40 ft. back from the exposed face of the seawall.

Settlement of the seawall would be of concern due to compression of the underlying clay from the weight of the wall and dike. It is not anticipated that the materials comprising the dike will compress to any significant degree. It is our opinion that the cost of the dike supported seawall would probably be less than the cost of the pile-supported seawall but more than a steel sheet pile bulkhead.

4. Stone Dike

A stone dike constructed from mudline to the proposed final grade is considered a practicable alternative. This structure would be similar to the dike that was previously discussed for support of the seawall

except that it would be constructed to final site grade (El. 17) and would be significantly wider at the base.

Assuming an average mudline at El. -25 along the dike, its height would be greater than 40 ft. The exposed dike face would be covered with placed granite armor stone from approximately El. -5 (below low tide level) to El. 17. Filter material would be required on the inboard side to preclude loss of fill material through the dike.

The inboard toe of the dike would extend approximately 80 ft. back from the top crest of the dike on the outboard side, resulting in a relatively large building set-back requirement. An advantage of this structure is that settlement of the dike would occur but would not be a major concern as a dike of this type could tolerate relatively large movements.

The stone dike would probably be the least costly of all alternatives discussed for the filled land area. (Refer to Figure 12).

D. Comparison of Shore Protection Systems for the Marina and Filled Land

As discussed in the previous sections, many factors must be considered in deciding the most appropriate shore protection system for the marina and in the area of the newly placed fill. In the marina area a vertical faced shore protection element (seawall or bulkhead) is considered to be the most appropriate solution because a wall facilitates boat mooring and water access. In the area to be filled, use of sloped shore protection system may be possible if boat moorings and water access are not intended in the development.

Based on the estimated costs, a steel sheet pile bulkhead in the marina will provide the most economical alternate. This alternate does have a basic disadvantage in that the steel sheeting is subject to corrosion and is considered to have a limited service life. Either the concrete diaphragm wall or precast panel wall would not be as susceptible to corrosion and would consequently have a longer life. Each of these wall types, however, are more expensive than a sheet pile bulkhead. The use of a

granite block seawall of the height required (27 ft.) is approximately two times more expensive than an anchor bulkhead wall. Refer to Figure 12 for a comparison of costs.

In the area to be filled the selection of the most feasible wall type is not as simple a decision as at the marina. Based soley on cost, the stone dike, faced with placed granite would be the favored alternative. This alternate, however, requires a building set-back from the top of the dike of approximately 80 ft. In addition, the sloped face impacts water related activities adjacent to the dike. Use of a granite block seawall supported on a stone dike or a steel sheetpile bulkhead will provide a vertical face adjacent to the water at approximately twice the cost.

V. FUTURE GEOTECHNICAL STUDIES

The preceding discussions in this report have provided general geotechnical considerations and preliminary recommendations relative to the Master Plan Development. Future studies will be required to collect additional subsurface information, and to develop definitive design criteria and final recommendations.

Future subsurface explorations will be required at building locations or other structure locations to provide the necessary data for design studies. In addition, the data is required to provide information to prospective contractors for their interpretation and evaluation as they prepare their cost estimates. Future explorations would include test borings, test pits and the installation of groundwater observation wells.

The geotechnical design studies conducted in the future are necessary to assess the following major geotechnical issues:

- temporary and permanent design soil and water loads;
- long and short term settlement of the compressible fill, organic soil and underlying marine clay;
- differential movements between various structural elements of the development;
- foundation type and design loading;
- aspects of dewatering and waterproofing;
- stability of existing and proposed shore protection elements; and,
- construction considerations related to the site work and foundation construction.

These studies should be conducted in coordination with design studies by other members of the project team so that the various design issues can be adequately assessed and the necessary geotechnical input provided to the project team.

VI. SUMMARY AND CONCLUSIONS

This report presents the results of our preliminary geotechnical studies for the Piers 1, 2, 3 development. The studies were based on the 1981 Master Plan Report prepared by HOK. The following is a summary of preliminary recommendations and conclusions developed during our studies.

A. Site Development

- Buildings located on the site must consider site features especially the existing seawalls and shore protection elements. It is recommended that buildings be set-back approximately 20 ft. from the existing seawalls and up to 90 ft. from the existing rubble dikes and sloped shore protection.

- Soil supported utilities should be located within the limits of the original Piers 1 and 2 filled during the 1800's.

- Pavements and final surface treatments will undergo future settlements and should be designed to accommodate these movements.

- The potential for loss of ground through the existing seawall is a concern along the portion of the site adjacent to water. The installation of filter material is recommended to reduce the potential for loss of ground.

B. Building Foundations

- Deep foundations (piles or caissons) are recommended for support of all proposed buildings with less than one basement level. Probable cost for these types of foundations at this site is approximately $30 per ton of supported load.

- Alternative foundation types (structural mat or individual spread footings) are feasible if multiple basement levels are incorporated into the project. The cost of these foundation types may be potentially less than deep foundations. The cost savings may be offset, however, by the cost of basement walls, floor slabs, waterproofing, excavation, temporary earth support and dewatering.

C. Excavation and Filling

- Excavation for the marina and below grade basement areas are considered technically feasible. The excavated material will probably require off-site disposal at an approved DEQE landfill because of the high chloride content of the excavated material.

- To create the proposed 2 acre land area, clean sand and gravel fill will probably be required from off-site borrow sources. If, following a review of the environmental requirements, use of on-site soils are permitted for filling, the filled area would be expected to undergo larger ground movements than if sand and gravel fill were utilized. Consequently, additional design requirements relative to building foundations and site development will be necessary if on-site fills are used.

- Placement of additional fill within the site area will result in long term consolidation of the clay, down drag forces on piles and additional premium foundation costs.

D. Shore Protection at the Marina

- In the proposed marina area, the most economical vertical shore protection system would be a steel sheet pile anchored bulkhead. This alternative is subject to corrosion and has a limited life. Use of concrete diaphragm wall or pre-cast panel wall bulkheads installed by the slurry trench method are viable alternatives, although more costly. These bulkhead alternatives should be considered, especially in consideration of their low corrosion susceptibility and longer life.

- Use of granite block seawalls in the marina area, if constructed from ground surface to dredge line, are expensive.

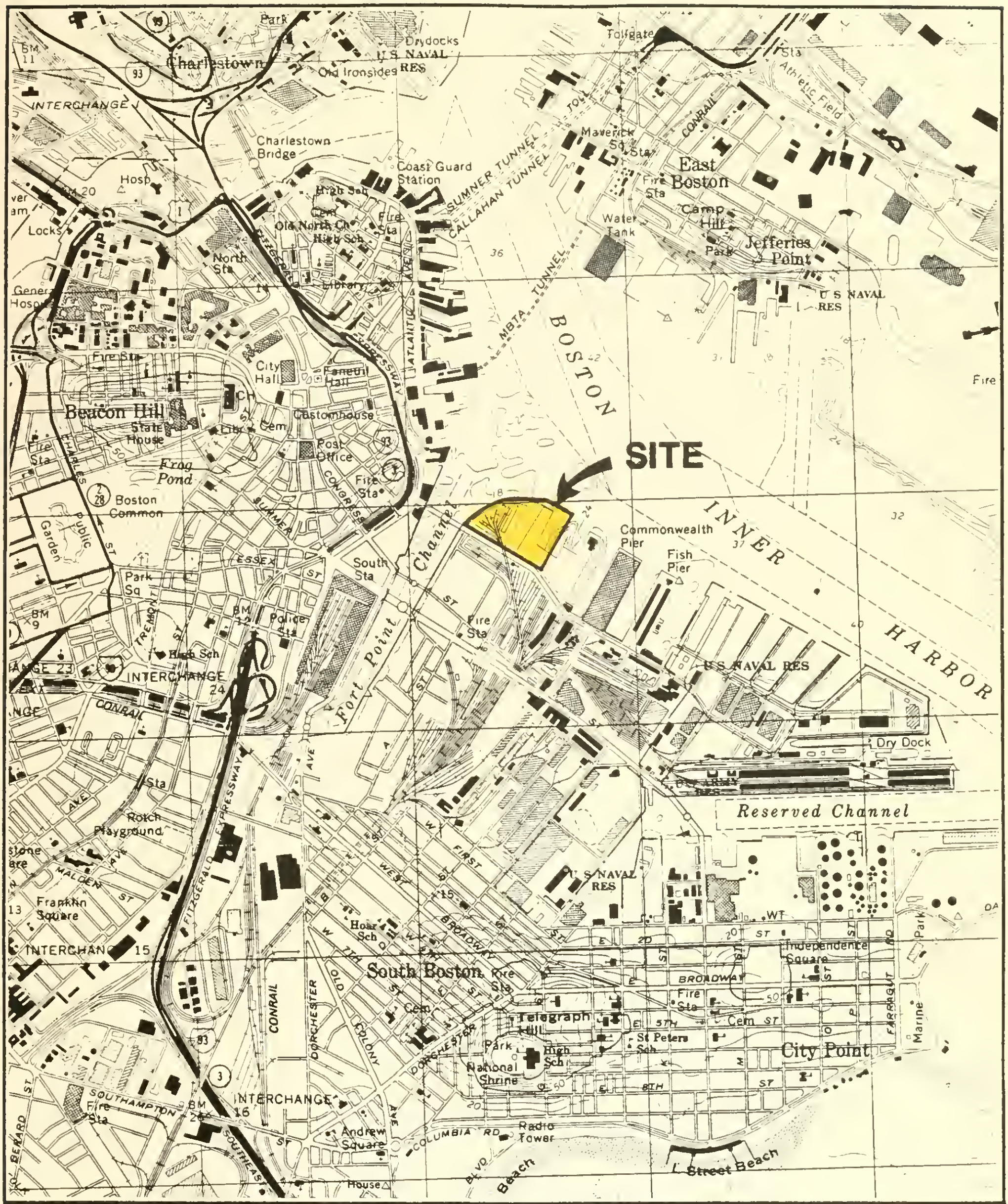
- Use of sloped shore protection in the marina area severely impacts boat moorings and water access along the shore unless a marginal wharf is constructed over the slope into deeper water.

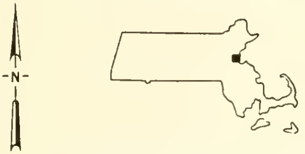
E. Shore Protection at the Perimeter of the Filled Land

- A rock dike constructed of dumped stone and faced with granite armor stone along the exposed slope is believed to provide the most economical shore protection system in this area. However, a set-back distance of about 80 ft. from the location of adjacent building foundations will be required. Water access and boat moorings will also be impacted by this type of shore protection.

- A steel sheet pile anchored bulkhead or a granite block seawall supported on a rock dike will provide a vertical face along the waters edge but at a much greater cost.

Figures





SITE COORDINATES: 331500E 4690900N

U.S.G.S. QUADRANGLE: BOSTON SOUTH, MA

HALEY & ALDRICH, INC.
CAMBRIDGE, MASSACHUSETTS
CONSULTING GEOTECHNICAL ENGINEERS, GEOLOGISTS AND HYDROGEOLOGISTS

HBC ASSOCIATES
FAN PIER DEVELOPMENT
SOUTH BOSTON, MASSACHUSETTS

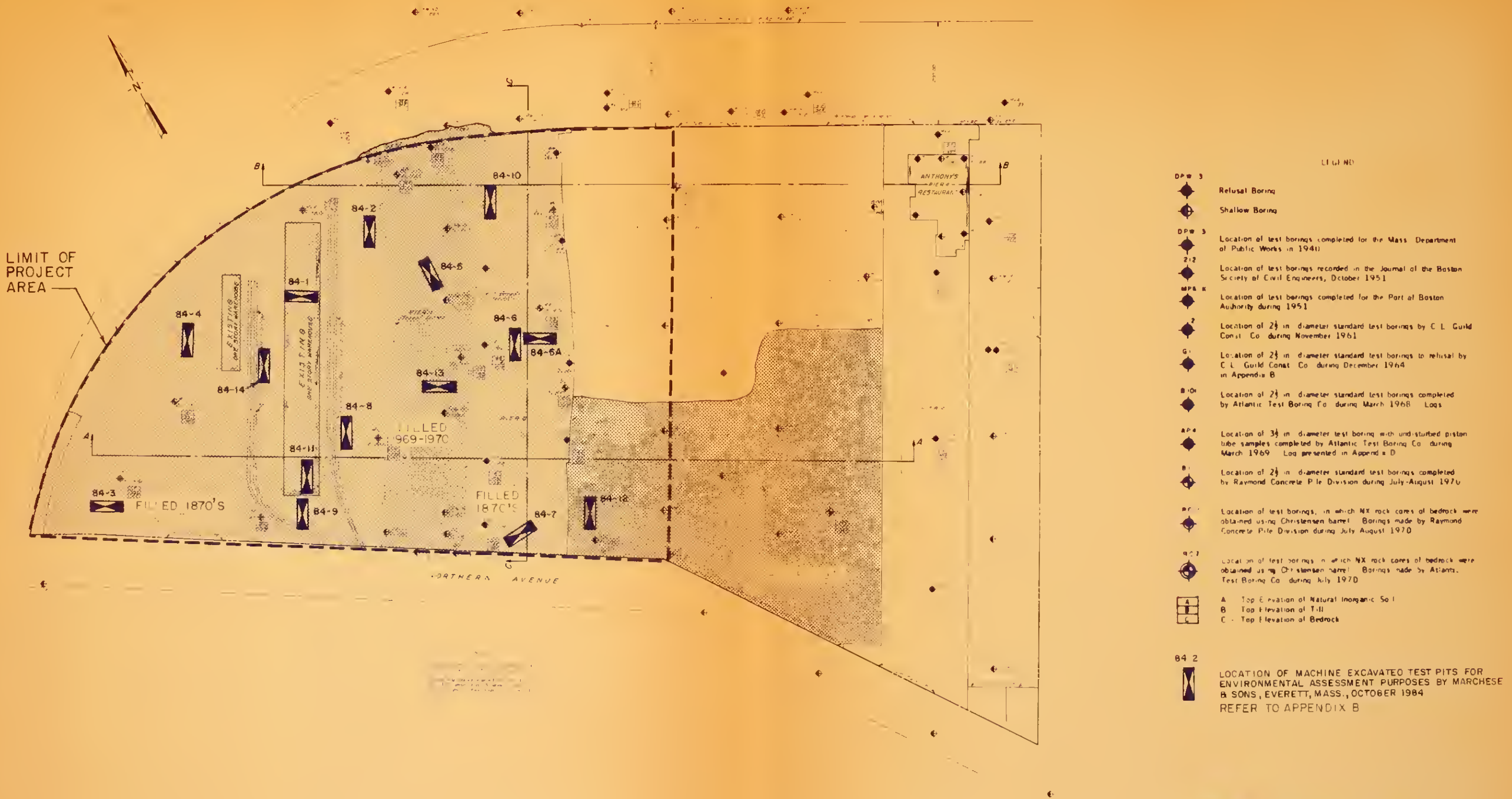
PROJECT LOCUS

APPROX. SCALE 1:25,000

JANUARY 1966

FILE NO. 5556 A22

FIGURE 1


N
LIMIT OF PROJECT AREA
B
B
A
A
C
C
84-10
84-2
84-5
84-1
84-4
84-6
84-6A
84-13
84-14
84-8
84-11
84-3
84-9
84-7
84-12
EXISTING ONE STORY WAREHOUSE
EXISTING ONE STORY WAREHOUSE
FILLED 1969-1970
FILLED 1870'S
FILLED 1870'S
ANTHONY'S PIER 4 RESTAURANT
NORTHERN AVENUE
LEGEND
DPW 3
Refusal Boring
Shallow Boring
DPW 3
Location of test borings completed for the Mass. Department of Public Works in 1940
212
Location of test borings recorded in the Journal of the Boston Society of Civil Engineers, October 1951
MPA K
Location of test borings completed for the Port of Boston Authority during 1951
2
Location of 2½ in. diameter standard test borings by C. L. Guild Const. Co. during November 1961
G1
Location of 2½ in. diameter standard test borings to refusal by C. L. Guild Const. Co. during December 1964 in Appendix B
B-101
Location of 2½ in. diameter standard test borings completed by Atlantic Test Boring Co. during March 1968. Logs
AP4
Location of 3½ in. diameter test boring with undisturbed piston tube samples completed by Atlantic Test Boring Co. during March 1969. Log presented in Appendix D
B1
Location of 2½ in. diameter standard test borings completed by Raymond Concrete Pile Division during July-August 1970
RC
Location of test borings, in which NX rock cores of bedrock were obtained using Christensen barrel. Borings made by Raymond Concrete Pile Division during July August 1970
RC7
Location of test borings in which NX rock cores of bedrock were obtained using Christensen barrel. Borings made by Atlantic Test Boring Co. during July 1970
A
B
C
A - Top Elevation of Natural Inorganic Soil
B - Top Elevation of Till
C - Top Elevation of Bedrock
84 2
LOCATION OF MACHINE EXCAVATED TEST PITS FOR ENVIRONMENTAL ASSESSMENT PURPOSES BY MARCHESE & SONS, EVERETT, MASS., OCTOBER 1984
REFER TO APPENDIX B

PLAN PREPARED FROM DATA
PRESENTED IN 1971 H&A REPORT

HALEY & ALDRICH, INC.
CAMBRIDGE MASSACHUSETTS
CONSULTING GEOTECHNICAL ENGINEERS AND GEOLOGISTS

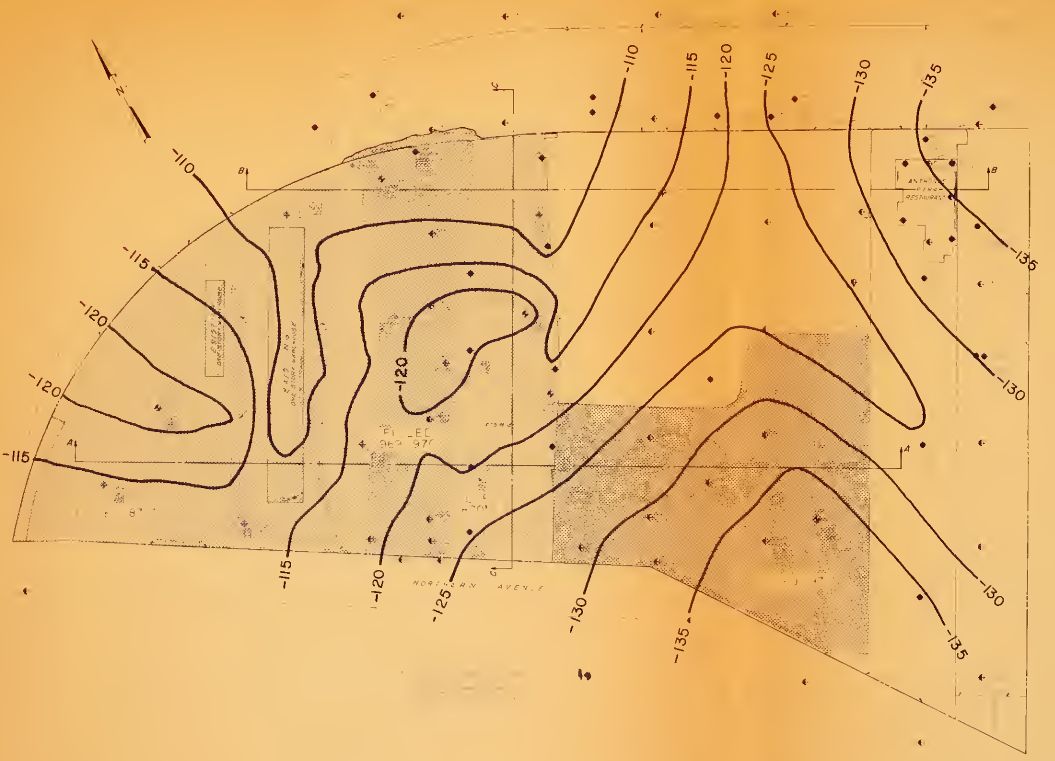
PIERS 1-2-3
MASTER PLAN DEVELOPMENT
BOSTON, MASSACHUSETTS

SITE AND SUBSURFACE EXPLORATION
LOCATION PLAN

SCALE: 1" = 200'
OCTOBER 1984

FIGURE 2

FILE NO. 5556 813


N
-110
-115
-120
-125
-130
-135
B
A
C
EXIST. ONE STORY WAREHOUSE
EXIST. ONE STORY WAREHOUSE
FILLED 1969-1970
NORTHERN AVENUE
ANTHONY'S PIER 4 RESTAURANT

PLAN PREPARED FROM DATA
PRESENTED IN 1971 H&A REPORT

HALEY & ALDRICH, INC
CAMBRIDGE MASSACHUSETTS
CONSULTING GEOTECHNICAL ENGINEERS AND GEOLOGISTS

PIERS 1-2-3
MASTER PLAN DEVELOPMENT
BOSTON, MASSACHUSETTS

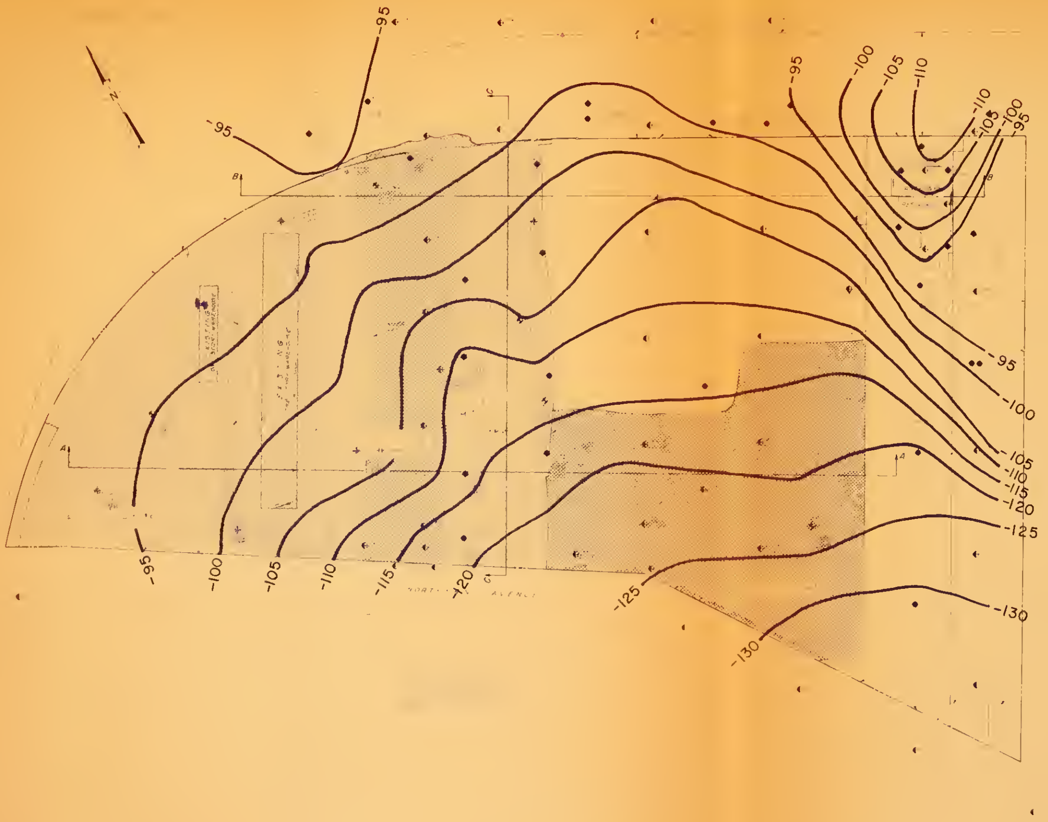
TOP OF BEDROCK CONTOUR PLAN

SCALE

OCTOBER 1984

FIGURE 3

FILE NO.



PLAN PREPARED FROM DATA
PRESENTED IN 971 H&A REPORT

HALEY & ALDRICH, INC
CAMBRIDGE MASSACHUSETTS
CONSULTING GEOTECHNICAL ENGINEERS AND GEOLOGISTS

[illegible]
[illegible] DEVELOPMENT
BOSTON, MASSACHUSETTS

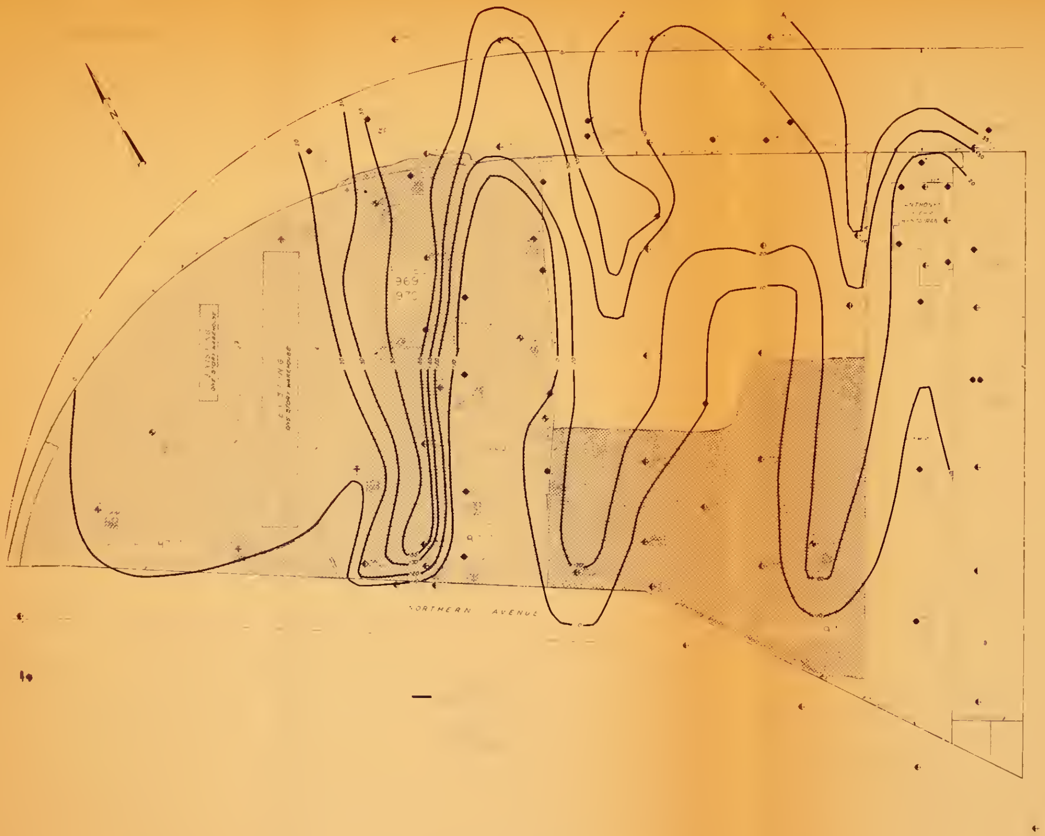
TOP OF GLACIAL TILL CONTOUR PLAN

SCALE [illegible]

OCTOBER 1984

FIGURE 4

FILE NO. 5556 84



PLAN PREPARED FROM [illegible] DATA
PRESENTED IN 1971 H&A REPORT

HALEY & ALDRICH, INC.
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CONSULTING GEOTECHNICAL ENGINEERS AND GEOLOGISTS

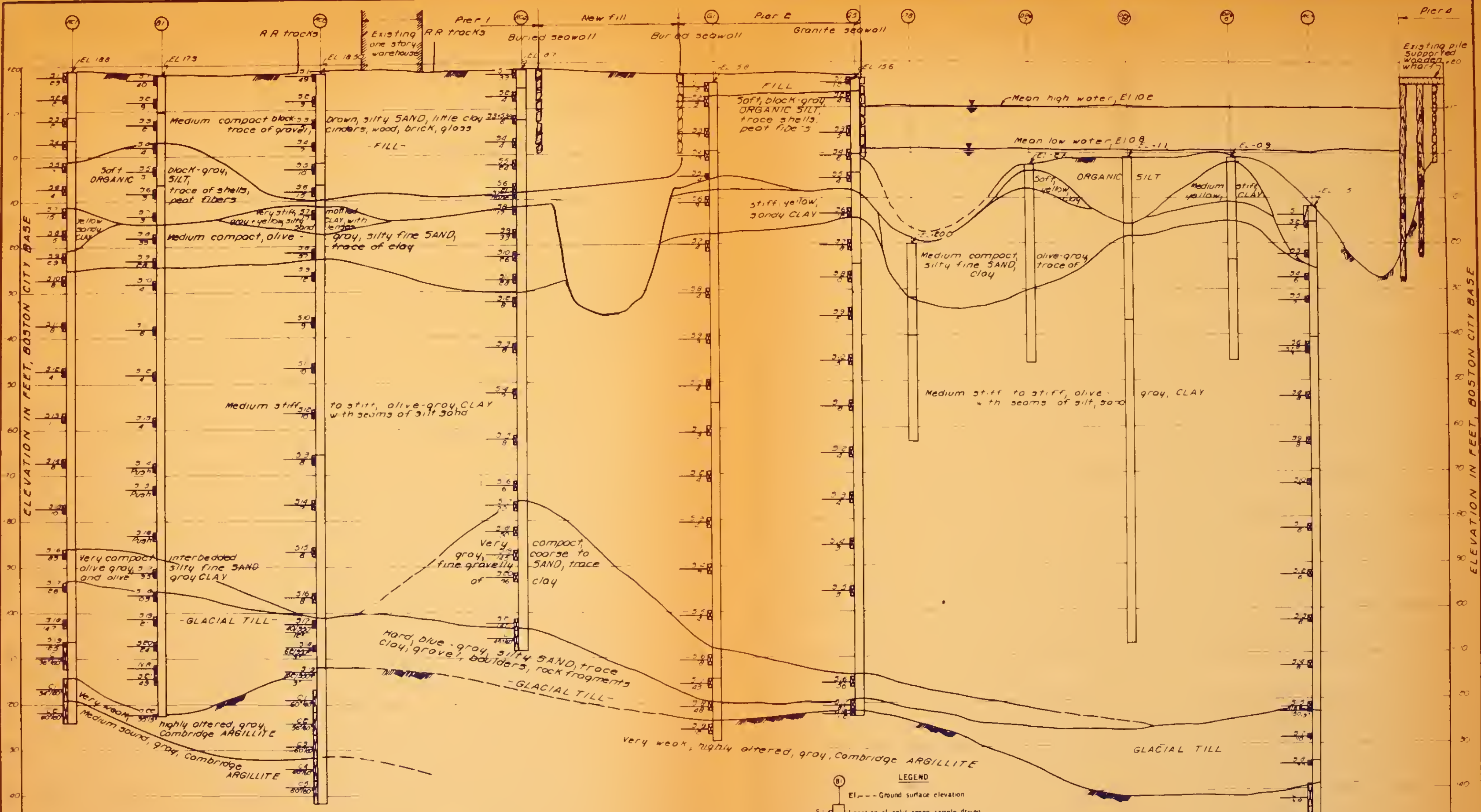
PIERS 1-3
MASTER PLAN DEVELOPMENT
BOSTON, MASSACHUSETTS

TOP OF INORGANIC SOILS CONTOUR PLAN

=200 OCTOBER 1984

FIGURE 5

FILE NO. 5556


RR tracks
Existing one story warehouse
RR tracks
Pier 1
Buried seawall
New fill
Buried seawall
Pier E
Granite seawall
Pier 4
Existing pile supported wooden wharf
EL 188
EL 179
EL 185
EL 87
EL 58
EL 156
Mean high water, El 10.0
Mean low water, El 0.0
FILL
Medium compact black trace of gravel
Brown, silty SAND, little clay cinders, wood, brick, glass
-FILL-
Soft ORGANIC
black-gray, SILT, trace of shells, peat fibers
Soft, black-gray ORGANIC SILT, trace shells, peat fibers
yellow sandy CLAY
very stiff, gray + yellow silty sand
mottled CLAY, with lenses
stiff, yellow, sandy CLAY
Medium compact, olive-gray, silty fine SAND, trace of clay
Soft, yellow clay
ORGANIC SILT
Medium yellow
stiff CLAY
Medium compact, silty fine SAND, clay
olive-gray trace of
Medium stiff to stiff, olive-gray CLAY with seams of silt sand
Medium stiff to stiff, olive-gray, CLAY with seams of silt, sand
Very compact olive gray and olive
interbedded silty fine SAND gray CLAY
-GLACIAL TILL-
Very compact, gray, fine gravelly coarse to SAND, trace of clay
Hard, blue-gray, silty SAND, trace clay, gravel, boulders, rock fragments
-GLACIAL TILL-
GLACIAL TILL
Very weak, highly altered, gray, Cambridge ARGILLITE
Medium sound, gray, Cambridge ARGILLITE
Very weak, highly altered, gray, Cambridge ARGILLITE
ELEVATION IN FEET, BOSTON CITY BASE
ELEVATION IN FEET, BOSTON CITY BASE
SOIL PROFILE A-A

LEGEND

El---Ground surface elevation

Location of split spoon sample driven with 140 lb hammer, 30 in drop, number indicates average blows per 12 penetration

Location of drilled NX rock core Core recovery 38' out of 60" drilled

NOTES

1. For location of borings and profiles see Figure 2
2. Elevation Datum is Boston City Base

WATERFRONT DEVELOPMENT
NORTHERN AVENUE
BOSTON, MASSACHUSETTS

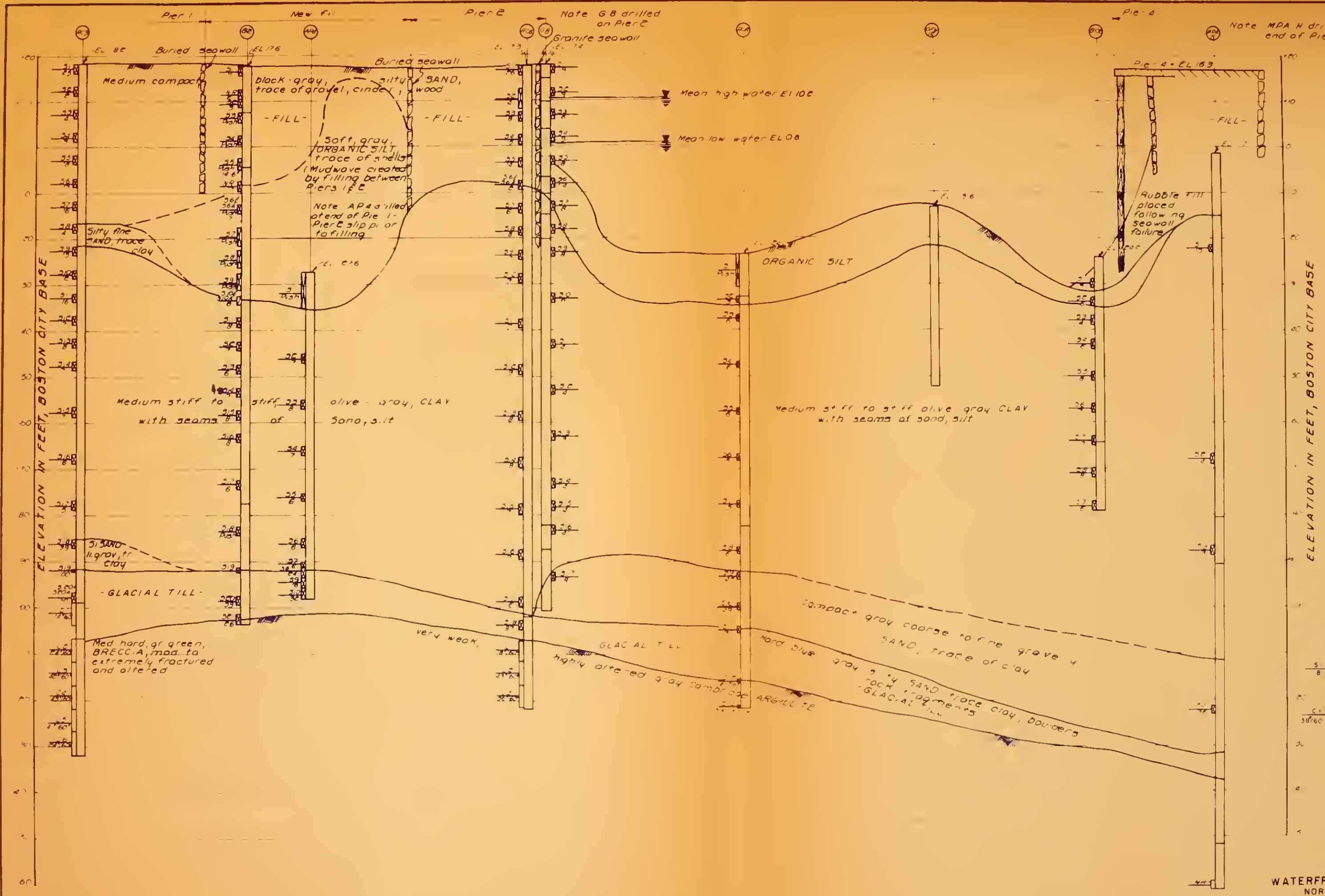
SOIL PROFILE A-A

Scales 0 10 20 30 50 Hor 0 10 20 Ver

October 1970
(Revised Jan 1971)

HALEY & ALDRICH, INC.
CONSULTING SOIL ENGINEERS

FIGURE 6


Pier 1
New fill
Pier 2
Note G8 drilled on Pier 2
Pier 4
Note MPA H drilled beyond end of Pier 4
Buried seawall
Granite seawall
Buried seawall
Medium compact
black-gray, silty SAND, trace of gravel, cinder, wood
-FILL-
-FILL-
Mean high water EL 10.0
Mean low water EL 0.8
Soft, gray, ORGANIC SILT, trace of shells
(Mudwave created by filling between Piers 1 & 2
Note AP4 drilled at end of Pier 1 - Pier 2 slip prior to filling
Silty fine SAND, trace clay
ORGANIC SILT
Pier 4 EL 16.3
-FILL-
Rubble fill placed following seawall failure
Medium stiff to stiff olive-gray CLAY with seams of sand, silt
Medium stiff to stiff olive gray CLAY with seams of sand, silt
Si SAND tr. grav, tr. clay
-GLACIAL TILL-
Med hard, gr green, BRECCIA, mod. to extremely fractured and altered
very weak,
highly altered gray cambridge ARGILLITE
GLACIAL TILL
Compact gray coarse to fine gravel SAND, trace of clay
Hard blue gray silty SAND trace clay, boulders rock fragments
-GLACIAL TILL-
ELEVATION IN FEET, BOSTON CITY BASE
ELEVATION IN FEET, BOSTON CITY BASE

SOIL PROFILE B-B

LEGEND

El - - - Ground surface elevation

S1 / 8 — Location of split spoon sample driven with 140 lb hammer, 30 in drop number indicates average blows per 12 penetration

C1 / 38/60 — Location of drilled NX rock core Core recovery 38 out of 60 drilled

NOTES

1 For location of borings and profiles see Figure 2

2 Elevation Datum is Boston City Base

WATERFRONT DEVELOPMENT
NORTHERN AVENUE
BOSTON, MASSACHUSETTS

SOIL PROFILE B-B

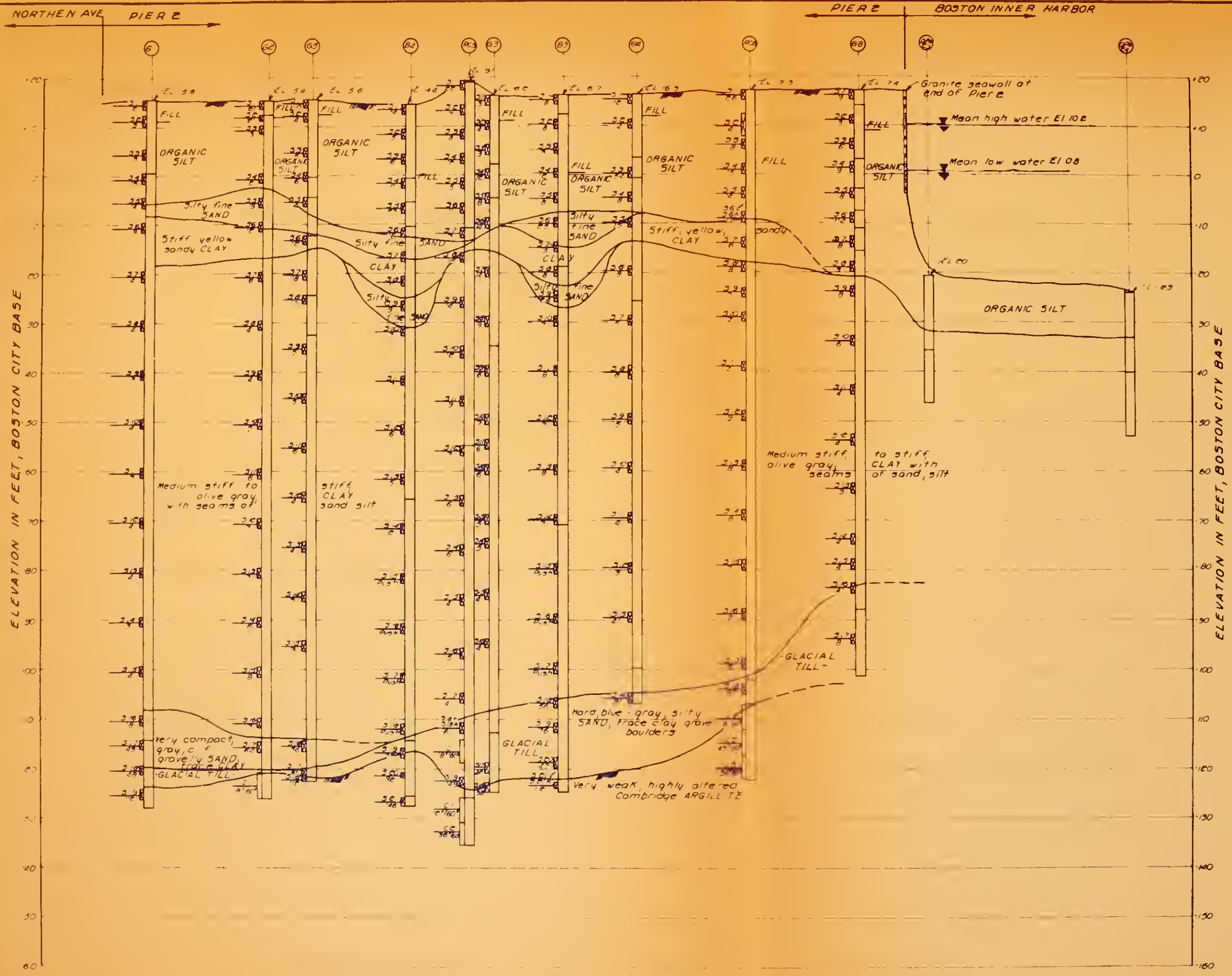
Scales: 0 10 20 30 50 Hor; 0 10 20 Ver

October 1970
(Revised Jan 1971)

FIGURE 7

HALEY & ALDRICH, INC.
CONSULTING SOIL ENGINEERS

FILE NO 5556 B15


NORTHEN AVE
PIER 2
PIER 2
BOSTON INNER HARBOR
ELEVATION IN FEET, BOSTON CITY BASE
FILL
ORGANIC SILT
Silty fine SAND
Stiff yellow sandy CLAY
Silty fine SAND
CLAY
Stiff, yellow, CLAY
sandy
Granite seawall at end of Pier 2
Mean high water El 10.2
Mean low water El 0.8
ORGANIC SILT
Medium stiff to olive gray with seams of
stiff CLAY sand silt
Medium stiff, olive gray, seams
to stiff CLAY with of sand, silt
GLACIAL TILL
Very compact, gray, c-f gravelly SAND, trace CLAY
GLACIAL TILL
Hard, blue-gray, silty SAND, trace clay grave boulders
GLACIAL TILL
Very weak, highly altered Cambridge ARGILLITE

SOIL PROFILE C-C

LEGEND

El. - - - - Ground surface elevation

Location of split spoon sample driven with 140 lb. hammer, 30 in. drop, number indicates average blows per 12" penetration.

Location of drilled NX rock core. Core recovery 38" out of 60" drilled.

NOTES

1. For location of borings and profiles see Figure 2
2. Elevation Datum is Boston City Base.
3. The stratification line between fill and organic silt is not shown on this profile Numerous changes have been made in these strata by dredging and filling between seawalls. Since the test borings represent conditions in both filled and dredgad zones, the stratification is too complex to be shown. Refer to logs for the individual borings and to Section III of the report text for further details.

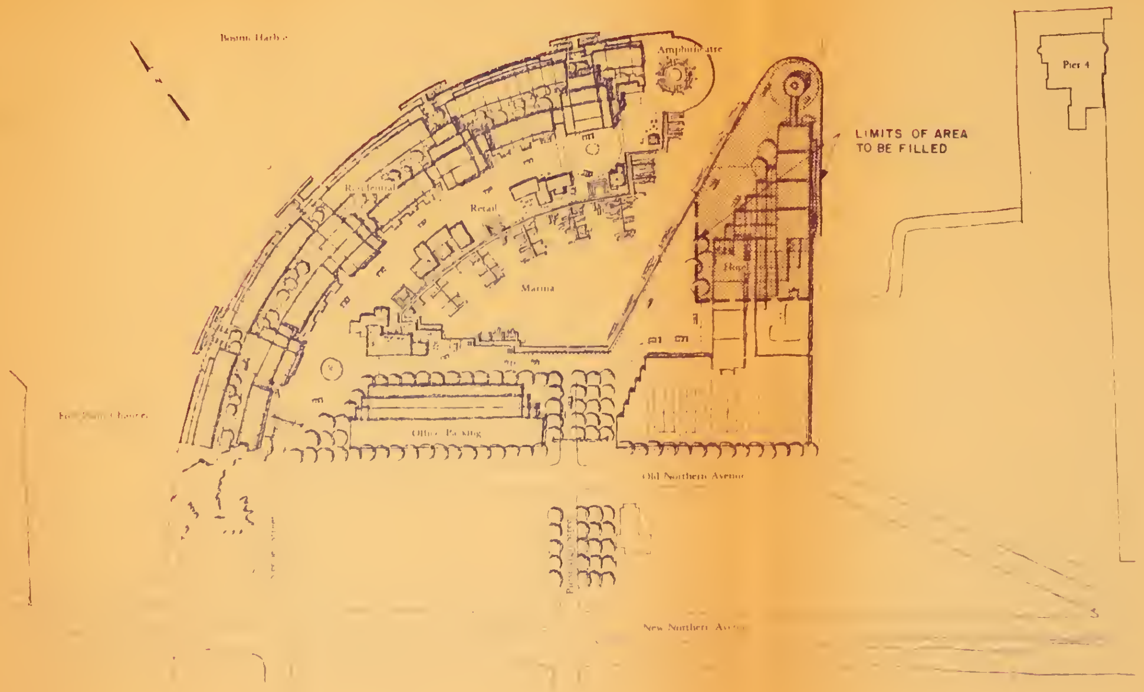
WATERFRONT DEVELOPMENT
NORTHERN AVENUE
BOSTON, MASSACHUSETTS
SOIL PROFILE C-C

Scales 0 10 20 30 60 Hor Ver

October 1970
(Revised Jan 1971)

HALEY & ALDRICH, INC.
CONSULTING SOIL ENGINEERS

FIGURE 8



NOTE:

THIS PLAN REPRODUCED FROM PLAN PREPARED BY HELLMUTH, OBATA & KASSABAUM, FOR THE MASTER PLAN REPORT DATED OCTOBER 1981.

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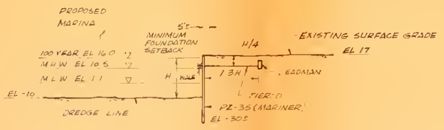
PIERS 1-3
MASTER PLAN DEVELOPMENT
BOSTON, MASSACHUSETTS

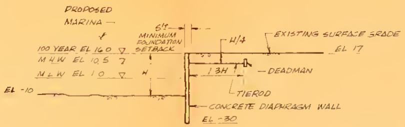
MASTER PLAN DEVELOPMENT

SCALE: 1"=200'
OCTOBER 1984

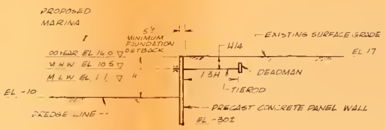
FIGURE 9

<table>
<tr><th rowspan="2">WALL TYPE
CONSTRUCTION CONDITIONS</th><th colspan="3">ANCHORED BULKHEADS</th></tr>
<tr><th>STEEL SHEET PILE</th><th>CONCRETE DIAPHRAGM WALL</th><th>PRECAST CONCRETE PANEL WALL</th></tr>
<tr><td>MARINA (ON LAND)</td><td>

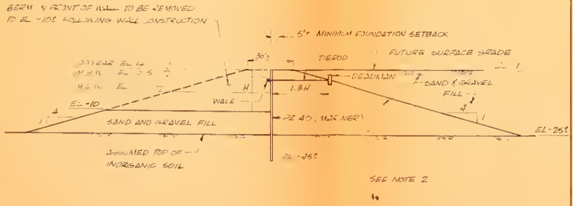
PROPOSED MARINA
5'±
MINIMUM FOUNDATION SETBACK
H/4
EXISTING SURFACE GRADE
EL 17
100 YEAR EL 16.0
MHW EL 10.5
MLW EL 1.1
H
WALE
1.3H
DEADMAN
TIEROD
EL -10
PZ-35 (MARINER)
DREDGE LINE
EL -30±

</td><td>

PROPOSED MARINA
5'±
MINIMUM FOUNDATION SETBACK
H/4
EXISTING SURFACE GRADE
EL 17
100 YEAR EL 16.0
MHW EL 10.5
MLW EL 1.0
H
1.3H
DEADMAN
TIEROD
EL -10
CONCRETE DIAPHRAGM WALL
EL -30

</td><td>

PROPOSED MARINA
5'±
MINIMUM FOUNDATION SETBACK
H/4
EXISTING SURFACE GRADE
EL 17
100 YEAR EL 16.0
MHW EL 10.5
MLW EL 1.1
H
1.3H
DEADMAN
TIEROD
EL -10
PRECAST CONCRETE PANEL WALL
DREDGE LINE
EL -30±

</td></tr>
<tr><td>FILLED LAND (IN WATER)</td><td>

BERM IN FRONT OF WALL TO BE REMOVED TO EL -10± FOLLOWING WALL CONSTRUCTION
5'± MINIMUM FOUNDATION SETBACK
30'±
TIEROD
FUTURE SURFACE GRADE
100 YEAR EL 16
MHW EL 10.5
MLW EL
DEADMAN
SAND & GRAVEL FILL
H
1.3H
WALE
EL -10
4
1
SAND AND GRAVEL FILL
PZ 40 (MARINER)
EL -25±
ASSUMED TOP OF INORGANIC SOIL
EL -45±
SEE NOTE 2

</td><td>NOT CONSIDERED APPLICABLE FOR CONSTRUCTION IN OPEN WATER</td><td>NOT CONSIDERED APPLICABLE FOR CONSTRUCTION IN OPEN WATER</td></tr>
</table>

NOTES:

1. THE TYPICAL SECTIONS INDICATED ON THIS FIGURE AND ON FIGURE 11 WERE DEVELOPED BASED ON AVERAGE SITE AND SUBSURFACE CONDITIONS. THE TYPICAL SECTIONS WERE USED TO DEVELOP COMPARATIVE COSTS. THE FINAL DESIGN OF ANY GIVEN STRUCTURE TYPE WILL NEED TO CONSIDER VARIATIONS IN EXISTING SITE AND SUBSURFACE CONDITIONS ALONG THE LINE OF THE STRUCTURE.
2. THE COST OF THE ANCHORED STEEL SHEET PILE BULKHEAD DOES NOT INCLUDE THE COSTS ASSOCIATED WITH FURNISHING BERM MATERIAL AND PLACING THE PORTION OF THE BERM BEHIND THE SHEET PILING. IT HAS BEEN ASSUMED THAT SUCH COSTS ARE CARRIED UNDER FILLING COSTS.
3. REFER TO FIGURES 11 AND 12 FOR ADDITIONAL NOTES REGARDING ASSUMPTIONS MADE IN DEVELOPING PROBABLE COSTS.

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CONSULTING GEOTECHNICAL ENGINEERS AND GEOLOGISTS

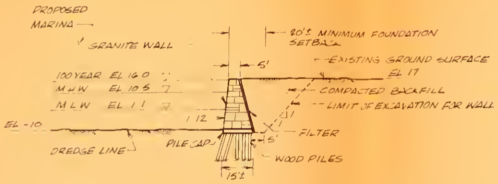
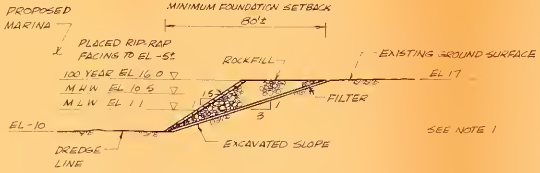
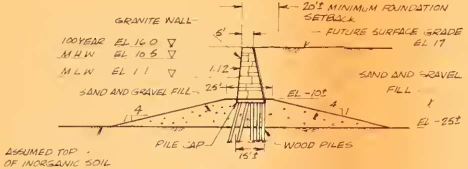
PIERS 1 2-3
MASTER PLAN DEVELOPMENT
BOSTON, MASSACHUSETTS

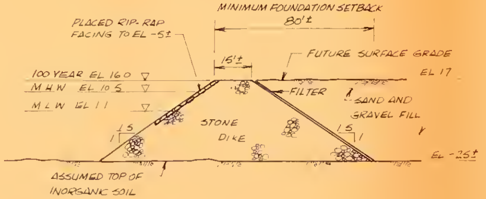
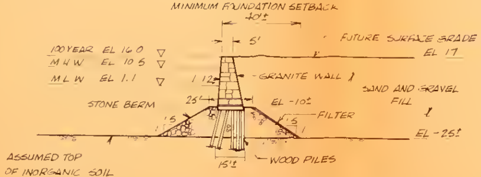
SCHEMATIC SECTIONS
SEAWALLS AND SLOPED SHORE PROTECTION

SCALE NONE
OCTOBER 1984

FIGURE 10

FILE NO 5556 011

<table>
<tr><th>WALL TYPE
CONSTRUCTION CONDITIONS</th><th></th><th>GRANITE BLOCK SEAWALL</th><th>SLOPED SHORE PROTECTION</th></tr>
<tr><td>MARINA (ON LAND)</td><td></td><td>
</td><td>
</td></tr>
<tr><td>FILLED LAND (IN WATER)</td><td>A</td><td>

PILE SUPPORTED SEAWALL</td><td>
</td></tr>
<tr><td></td><td>B</td><td>

STONE DIKE SUPPORTED SEAWALL</td><td></td></tr>
</table>

NOTES:

1. THE COST OF SLOPED SHORE PROTECTION FOR THE MARINA INCLUDES EXCAVATION OF EXISTING MATERIAL BETWEEN A VERTICAL LINE AT THE TOE OF THE SLOPE TO THE 3H TO 1V LINE.
2. REFER TO FIGURES 10 AND 12 FOR ADDITIONAL NOTES REGARDING ASSUMPTIONS MADE IN DEVELOPING PROBABLE COSTS.

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BOSTON, MASSACHUSETTS

SCHEMATIC SECTIONS
SEAWALLS AND SLOPED SHORE PROTECTION

SCALE NONE
OCTOBER 1984

FIGURE 11

FACTOR	STEEL SHEET PILE ANCHORED BULKHEAD		CONCRETE DIAPHRAGM WALL ANCHORED BULKHEAD		PRECAST CONCRETE PANEL ANCHORED BULKHEAD		GRANITE BLOCK SEAWALL		SLOPED SHORE PROTECTION	
	MARINA	FILLED LAND	MARINA	FILLED LAND	MARINA	FILLED LAND	MARINA	FILLED LAND	MARINA	FILLED LAND
CONSTRUCTABILITY	DRIVING OF STEEL SHEET PILING FEASIBLE. OBSTRUCTION REMOVAL EXPECTED TO BE REQUIRED AT SOME LOCATIONS.	ADVANCE SOIL BERM TO PERMIT PILE INSTALLATION. SAFE OPERATION OF EQUIPMENT ON AND STABILITY OF BERM PRIMARY CONSTRUCTION CONSIDERATIONS.	USE OF SLURRY TRENCH METHOD FEASIBLE. OBSTRUCTION REMOVAL EXPECTED TO BE REQUIRED AT SOME LOCATIONS.	CONSTRUCTION NOT CONSIDERED PRACTICABLE. MAINTAINING STABLE SLURRY TRENCH IN LOOSE BERM FILL NOT CONSIDERED FEASIBLE.	SAME AS CONCRETE DIAPHRAGM WALL.	SAME AS CONCRETE DIAPHRAGM WALL.	CONSTRUCTION FEASIBLE.	CONSTRUCTION FEASIBLE. PILE-SUPPORTED AND STONE DIKE SUPPORTED SEAWALLS CONSIDERED.	CONSTRUCTION FEASIBLE. ON LAND CONSTRUCTION WOULD CONSIST OF SLOPED SHORE PROTECTION	CONSTRUCTION FEASIBLE. IN WATER CONSTRUCTION WOULD CONSIST OF STONE DIKE
LOCAL CONSTRUCTION PRACTICE	COMMONLY USED FOR ANCHORED BULKHEADS		NO KNOWN LOCAL MARINE CONSTRUCTION OF THIS WALL TYPE.	—	NO KNOWN LOCAL MARINE CONSTRUCTION OF THIS WALL TYPE.	—	COMMONLY USED IN PAST. NOT COMMONLY USED TODAY.		COMMONLY USED	COMMONLY USED
[illegible]	CORROSION MOST SEVERE IN SPLASH ZONE. LIMITED LIFE SPAN. USE OF MARINER STEEL CAN EXTEND LIFE.		MINIMAL CORROSION. CORROSION RESISTANT CONCRETE MIX DESIRABLE	—	MINIMAL CORROSION. CORROSION RESISTANT CONCRETE MIX DESIRABLE.	—	NONE		NONE	
ENVIRONMENTAL CONSIDERATIONS	NOISE FROM SHEET PILE DRIVING	NOISE FROM SHEET PILE DRIVING AND FILLING IN WATER TO CONSTRUCT BERM.	NONE	—	NONE	—	NONE	FILLING IN WATER TO CONSTRUCT BERM.	NONE	FILLING IN WATER TO CONSTRUCT BERM.
[illegible]	EXPOSED WALL SURFACE CONSISTS OF STEEL WHICH WILL RUST.		EXPOSED WALL SURFACE CONSISTS OF ROUGH AND IRREGULAR CONCRETE.	—	EXPOSED WALL SURFACE CONSISTS OF SMOOTH CONCRETE.	—	EXPOSED SURFACE CONSISTS OF [illegible]		EXPOSED SURFACE CONSISTS OF GRANITE.	
[illegible]	[illegible]		SAME AS STEEL SHEET PILE BULKHEAD.	—	SAME AS FOR STEEL SHEET PILE BULKHEAD.	—	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]		[illegible]	—	SAME AS STEEL SHEET PILE BULKHEAD.	—	[illegible]		CONSTRUCTION OF A PILE SUPPORTED [illegible] OVER SLOPE REQUIRED FOR [illegible] VESSELS.	
[illegible]	[illegible]	[illegible] CLAY.	[illegible]	—	ANTICIPATED TO PERFORM WELL.	—	ANTI[illegible]	[illegible] CLAY.	ANTICIPATED TO PERFORM WELL.	[illegible] CLAY.
[illegible]	$1[illegible] - $1[illegible]	$4[illegible] (SEE NOTE [illegible])	[illegible]	—	$1[illegible]00 - $[illegible]	—	$4[illegible] - $4[illegible]	$[illegible] - $5[illegible] (PILE SUPPORTED) $4[illegible] - $4[illegible] (STONE DIKE SUPPORTED)	$1100 - $1200 (SEE NOTE 4) $2800 - $3000 (WITH WOOD PILE SUPPORTED DECK)	$1500 - $1[illegible]
[illegible]	$[illegible],000	$3,9[illegible]	$4,[illegible]	—	$4,50[illegible]	—	$4,5[illegible],000	$5,[illegible] (PILE SUPPORTED) $4,2[illegible] (STONE DIKE SUPPORTED)	$3,[illegible],000 $6,[illegible],000 (WITH WOOD PILE SUPPORTED DECK)	$1,[illegible]

NOTES:

1. THE PROBABLE COSTS INDICATED UNDER ITEM I WERE DEVELOPED ON THE BASIS OF TYPICAL SECTIONS OF THE STRUCTURE ALTERNATIVES DEPICTED ON FIGURES 10 AND 11. THE TYPICAL SECTIONS INDICATED ARE FOR AVERAGE SITE AND SUBSURFACE CONDITIONS. THE FINAL DESIGN OF ANY GIVEN STRUCTURE TYPE WILL NEED TO CONSIDER VARIATIONS IN EXISTING SITE AND SUBSURFACE CONDITIONS ALONG THE LINE OF THE STRUCTURE.

2. TOTAL PROBABLE COSTS INDICATED UNDER ITEM J INCLUDE THE FOLLOWING:

   MARINA:

   A. CONSTRUCTION OF A BULKHEAD, SEAWALL OR SLOPED SHORE PROTECTION AT THE PERIMETER OF THE MARINA AND EXCAVATION AND DISPOSAL (AT A MARINE LANDFILL) OF EXISTING SOILS WITHIN THE LIMITS OF THE MARINA.

   B. A MARINA PLAN AREA OF 3 ACRES.

   C. A MARINA DEPTH OF 27 FT. (EL. 17 TO EL. -10).

   D. A MARINA PERIMETER OF 1,700 FT.

   E. TOTAL EXCAVATION VOLUME OF 130,000 CUBIC YARDS CALCULATED BASED ON B, C AND D ABOVE.

   FILLED LAND:

   A. CONSTRUCTION OF A BULKHEAD, SEAWALL OR SLOPED SHORE PROTECTION AT THE PERIMETER OF THE FILLED LAND AREA.

   B. A FILLED LAND AREA OF [illegible] ACRES.

   C. AN AVERAGE FILL DEPTH OF 40 FT.

   D. A FILLED LAND PERIMETER OF 800 FT.

   E. TOTAL FILL VOLUME OF 130,000 CUBIC YARDS (CY) OF GRANULAR FILL CALCULATED ON THE BASIS OF B, C AND D ABOVE.

   F. WHEN ESTIMATING REQUIRED FILL VOLUMES FOR THE BULKHEAD, SEAWALL AND SLOPED SHORE PROTECTION ALTERNATIVES, THE BASE FILL VOLUME (130,000 CY) WAS REDUCED TO ACCOUNT FOR FILL VOLUMES AND WALL VOLUMES LOCATED BEHIND A WALL LINE CHOSEN AS FOLLOWS:

      i. FOR GRANITE BLOCK SEAWALLS, WALL LINE AT EXPOSED FACE OF SEAWALL. BASE FILL VOLUME REDUCED BY VOLUME OF WALL AND SOIL OR STONE BERM BEHIND WALL LINE.

      ii. FOR SLOPED SHORE PROTECTION (STONE DIKE), WALL LINE AT OUTBOARD SIDE OF CREST OF DIKE. BASE FILL VOLUME REDUCED BY VOLUME OF DIKE BEHIND WALL LINE.

      iii. FOR STEEL SHEET PILE BULKHEAD, NO CHANGE IN BASE FILL VOLUME. SEE NOTE 3.

3. THE COST OF THE ANCHORED STEEL SHEET PILE BULKHEAD FOR THE PROPOSED FILL AREA SHOWN ON FIGURE 10 DOES NOT INCLUDE COSTS OF FURNISHING BERM MATERIAL AND PLACING THE PORTION OF THE BERM BEHIND THE SHEET PILING. SUCH COSTS HAVE BEEN INCLUDED UNDER FILLING COSTS.

4. THE COST OF SLOPED SHORE PROTECTION FOR THE MARINA INDICATED ON FIGURE 11 INCLUDES EXCAVATION OF EXISTING MATERIAL BETWEEN A VERTICAL LINE AT THE TOE OF THE SLOPE TO THE 5H TO 1V LINE.

5. PROBABLE CONSTRUCTION COSTS DO NOT INCLUDE GENERAL CONTRACTOR MARK UP AND CONSTRUCTION CONTINGENCIES. IT IS RECOMMENDED THAT A [illegible] PERCENT COST CONTINGENCY BE CARRIED WHEN DEVELOPING BUDGET [illegible]

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COMPARISONS OF ANCHORED BULKHEADS,
SEAWALLS AND SLOPED SHORE PROTECTION

NO SCALE
OCTOBER 1984

FIGURE 12

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